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DISCUSSION

(over)

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Proceedings of the American Society of Civil Engineers

TESTS ON A 120-FT-SPAN PRESTRESSED CONCRETE BEAM

By G. D. Base[1] and R. E. Rowe[2]

SYNOPSIS

Tests to destruction have been carried out on a 120-ft-span prestressed concrete beam similar to the beams to be used in the construction of a four-span overpass at the Chiswick rotary junction. The tests showed that the design assumptions with regard to the loss of stress due to friction and short-term creep, shrinkage, and relaxation were valid. The behaviour of the beam in the working-load range agreed excellently with the behaviour predicted.

At ultimate load the beam failed by crushing of the concrete in the top flange at a deflection of 20 in. The calculated ultimate load was 6% less than the actual failing load; the assumptions, on which the ultimate load calculations were based, were thus fully vindicated.

INTRODUCTION

At the western end of the Cromwell Road extension scheme there is an overpass, consisting of four spans each of about 125 ft, carrying dual 24 ft roadways over a rotary junction at the junction of the Great West Road, North Circular Road, and Chiswick High Road. The Ministry of Transport, for whom the overpass was being constructed, specified that a beam similar to the beams used in the bridge should be tested to destruction.

DETAILS OF THE BEAM AND THE TESTING RIG

The beam, which was 120 ft in span and had an over-all depth varying from 72 in. at mid-span to 81 in. at the supports, consisted of three precast sections which were jointed, post-tensioned, and provided with an in situ top flange.

Note.—Discussion open until February 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. ST 9, September, 1960.

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The center precast section was 47 ft 9 in. long with a cross-section at mid-span as shown in Fig. 1; the bottom flange of the unit increased in depth from 10 in. at mid-span to $11\frac{1}{2}$ in. at the ends. The end precast sections were 38 ft $3\frac{1}{2}$ in. long. Each section was post-tensioned with a number of secondary cables (Freyssinet). The eighteen straight $1\frac{3}{4}$ in. diam ducts for the main post-tensioning cables were formed in the bottom flange of each unit by casting in steel sheathing.

The concrete for the precast sections contained 3/4 in. Enderby granite, 3/4 in. Curtis shingle, 1/2 in. Tuckwell shingle, Curtis sharp sand, and ordinary Portland cement; the aggregate-cement ratio was 3.23 and the water-cement ratio was 0.37. The strengths of the concrete, as found from tests on 6 in. cubes, are given in Table 1.

The precast sections of the beam were erected in the test rig on supports identical with those to be used in the actual overpass; one end was supported on a Meehanite rocker (Fig. 2) and the other on a Meehanite hinge (Fig. 3). The units were temporarily supported at the correct level at the section of the joints and provision was made for movement of the units as the post-tensioning was applied. The joints, which were 3 in. wide and provided with short lengths of rubber tube to maintain the continuity of the ducts, were then concreted. The concrete used in the joints consisted of 3/8 in. to 3/16 in. Wraysbury gravel, Wraysbury sand, and rapid-hardening Portland cement; the aggregate-cement ratio was 2.5 and the water-cement ratio 0.32. The 6 in. cube strengths for this concrete are given in Table 1.

Five days after the joints had been cast post-tensioning of the main cables was commenced. There were eighteen Freyssinet cables, each with twelve 0·276 in. diam wires. The anchorages were external to the beam and the female cones thrust directly on a steel plate $1\frac{1}{2}$ in. thick cast in the unit (Fig. 2). The average properties of the high-tensile steel wire used in stressing the beam were as follows: the diameter was 0.276 in., the ultimate tensile strength 232,000 lb per sq in., the 0.2% proof stress 207,000 lb per sq in., and the Young's modulus 28.5×10^6 lb per sq in. These properties were deduced from the test results obtained on three short lengths cut from each of the coils used. A typical load-strain curve for the wire is shown in Fig. 4. Each cable was stressed from one end only with an initial force per cable of 113 kips. The average force per cable, after allowance has been made for the progressive elastic loss during stressing, was 109 kips.

Grouting of the cables was carried out immediately after all the cables had been stressed; the grout used was of neat cement with a water-cement ratio of 0.5.

The in situ top flange, 13 in. thick, was cast in three sections. The central section was cast first and then the two end sections were cast after a period of five days. The concrete used in the flange consisted of 3/4 in. Curtis shingle, 1/2 in. Tuckwell shingle, Curtis sharp sand, and ordinary Portland cement; the aggregate-cement ratio was 5 and the water-cement ratio 0.45. The details of the quality control and the crushing strengths of cubes and cylinders for the concrete of the top flange are given in Table 2.

A general view of both the beam and the test rig is shown in Fig. 5. Details of the two loading frames are shown in Fig. 6. The test rig consisted of reinforced concrete foundation blocks below ground level to which the 18 in. x 18 in. concrete columns and the 4 ft x 1 ft 6 in. reinforced transoms were

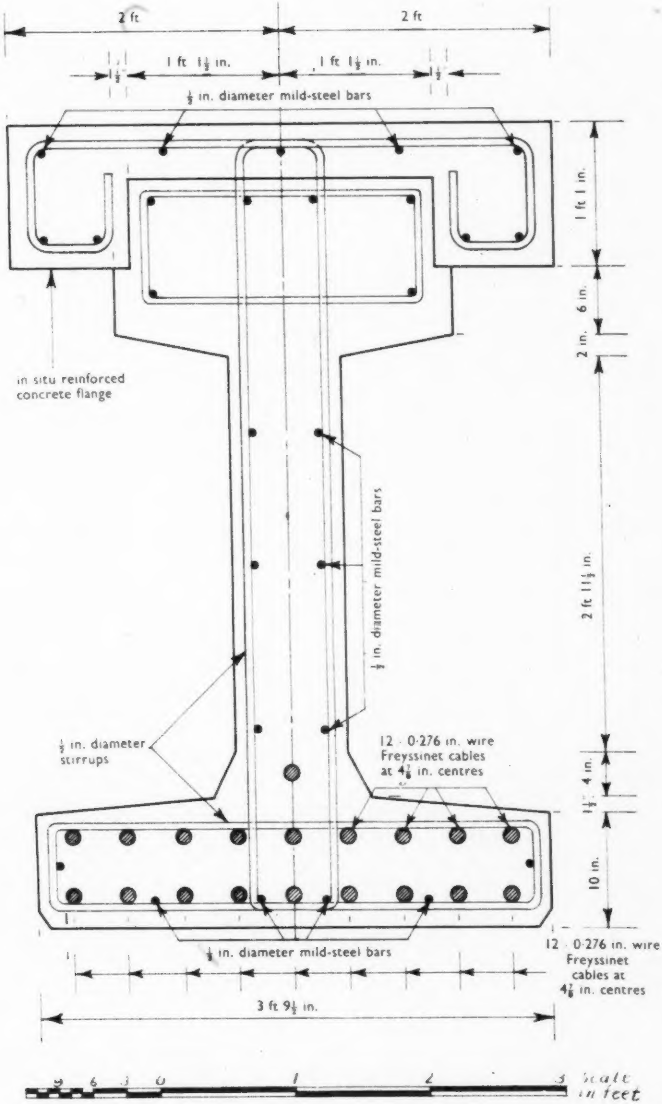

FIG. 1.—CROSS SECTION OF BEAM AT MID-SPAN

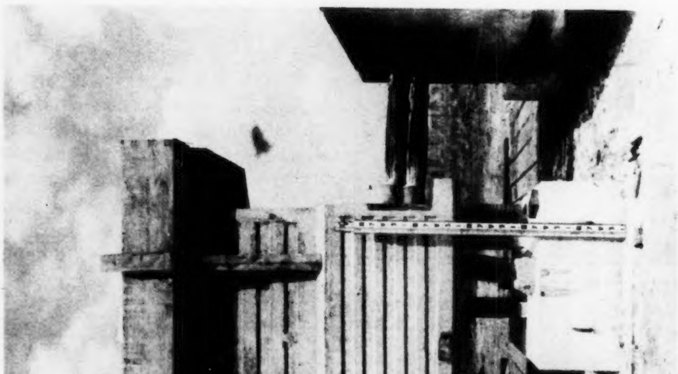

FIG. 2.—ROCKER SUPPORT

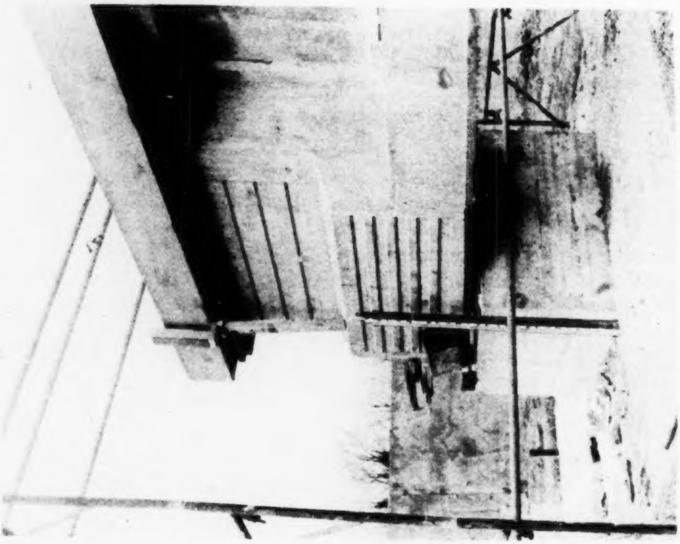

FIG. 3.—HINGE SUPPORT

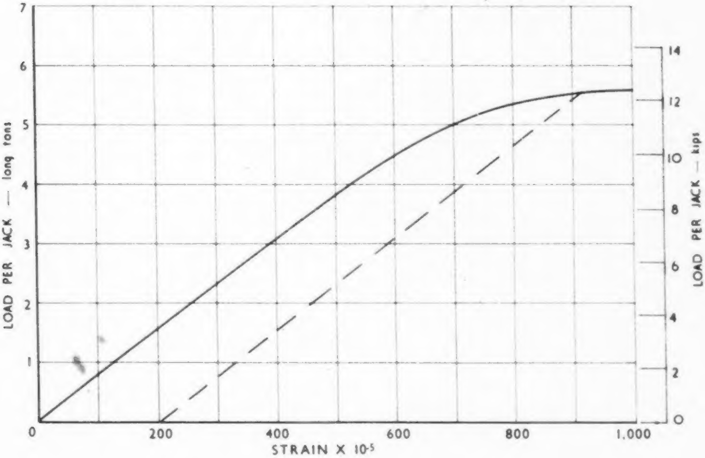

FIG. 4.—LOAD-STRAIN CURVE FOR HIGH TENSILE STEEL

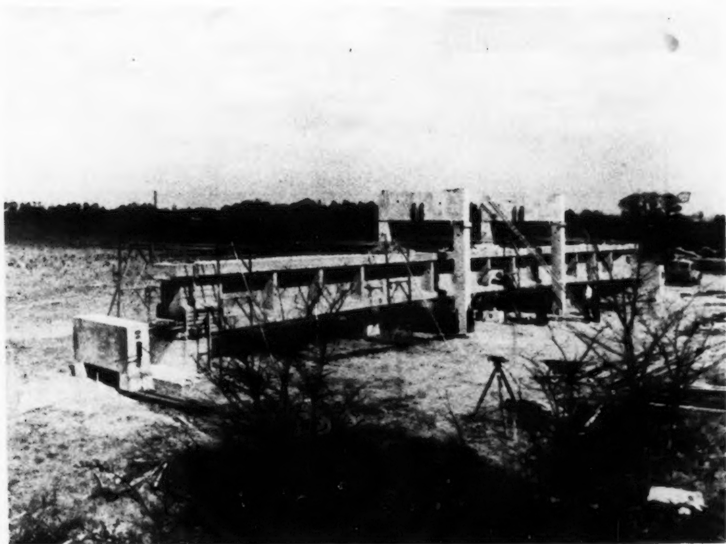

FIG. 5.—VIEW OF THE BEAM AND TEST RIG

TABLE 1.—CUBE STRENGTHS AT VARIOUS AGES FOR THE PRECAST SECTIONS OF THE BEAMS AND THE JOINTING CONCRETE

Unit (1)	End 1 (2)			Center (3)			End 2 (4)		
(a) Precast sections									
Age, in days	10	28	169[a]	10	28	159[a]	10	28	165[a]
Cube strength, in lb per sq in.	8,300	8,380	9,120	8,340	9,460	9,870	8,300	8,930	9,940

(b) Jointing concrete			
Age, in days	4	14	28
Cube strength, in lb per sq in.	6,500	7,800	8,290

[a] At time of test.

TABLE 2.—DETAILS OF QUALITY CONTROL AND CRUSHING STRENGTH OF CONCRETE USED IN IN SITU TOP FLANGE

(a) Reference numbers of test specimens

Date cast	Position	Sample Number	Compacting factor	Cubes	Cylinders (cast upright)	Cylinders (cast on side)
March 4, 1958	center section	1	0.89	1A	1B	2B
		2	0.87	2A,3A		3B,4B
August 4, 1958	anchorage end section	1	0.75	4A,5A		6B,7B
		2	0.86	6A	5B	8B
August 4, 1958	jacking end section	1	0.90	7A,8A		9B,10B
		2	0.85	9A	12B	11B

(b) Results of strength tests

Specimens	Age at test		
	7 days	12 days	At test of beam[a]
6 in. cubes	1A 3,690 4A 5,020 7A 3,440 6A 4,220 9A 3,390	3A 4,690	5A 4,170 8A 5,750 2A 6,120*
cylinders	8B 4,310 9B 3,280	3B 3,730	7B 5,370 11B 4,640
cylinders (sulphur capped)	5B 4,240 12B 3,850		1B 4,740
cylinders (tension)	3B 318 6B 380 10B 355	4B 330	

[a] 14 days, except where marked with an asterisk which was tested at 19 days.

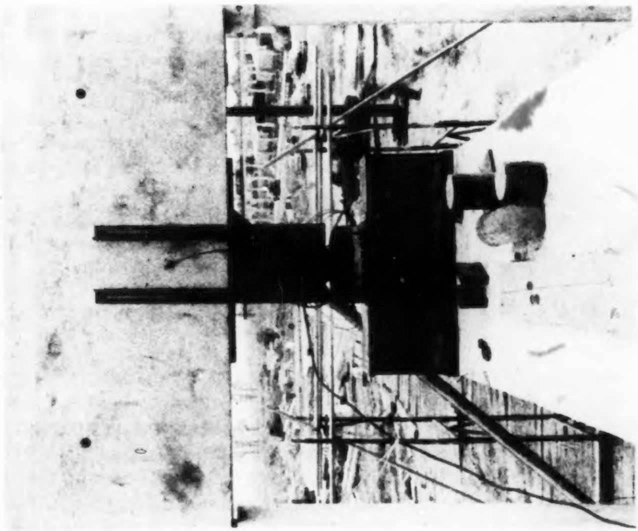

FIG. 7.–LOADING ARRANGEMENT

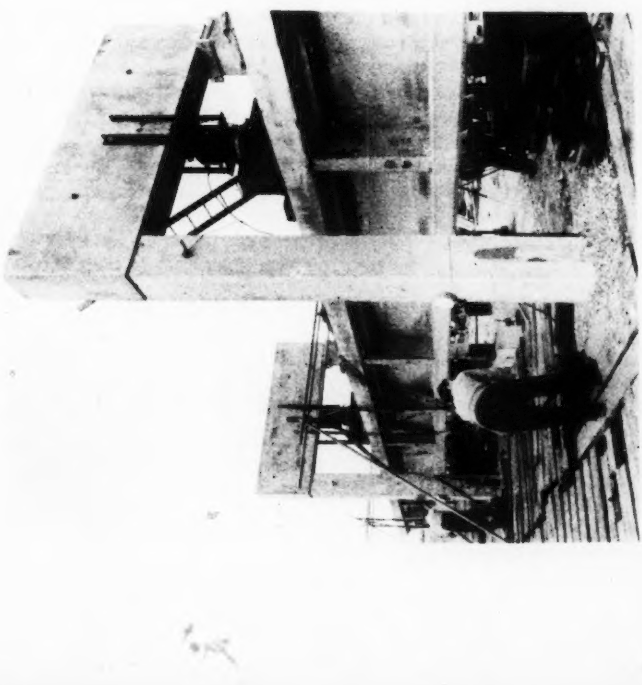

FIG. 6.–LOADING FRAMES

post-tensioned with two Gifford-Udall cables per column, each cable consisting of twelve 0.276 in. diam wires. The size of the foundation blocks was governed by the estimated ultimate load for the beam and the density of the back filling. The bearing for each transom beam consisted of an 18 in. x 6 in. x 3/4 in. steel plate bedded on the column with mortar. The two portal frames were at 30 ft centres and were equidistant from the centre of the beam. The end supports for the beam consisted of mass concrete blocks 15 ft square with plinths to give a clearance of 3 ft under the beam at the supports.

In the first series of tests, load was applied to the beam by hydraulic jacks, each 330 kips in capacity and with 6 in. travel; these were attached by cradles to each transom and operated by hand pumps. Each load was applied through a 330 kips load capsule to a spreader beam which was separated from the top flange of the beam by pads of rubber 2 in. thick. A detail of the loading arrangement is given in Fig. 7. It was necessary to pack the jacks and this was done by using two Tangye Hydralite jacks, one on either side of the main jacks. In the final test electrically operated hydraulic jacks with 12 in. travel were used but even these had to be packed twice before the beam failed.

DEFLECTION AND STRAIN MEASURING EQUIPMENT

When the precast sections were stressed together the longitudinal shortening and the hogging of the beam were measured by 0.001 in. dial gages bearing on the ends of the units and on the soffit at mid-span. During the loading tests deflections were obtained at the quarter- and mid-span sections with vernier wire gages reading to 0.01 in. and at mid-span with 0.001 in. dial gages; these gages were mounted on both sides of the beam. Additional dial gages were used to determine any settlement at the supports. Two levels were used to determine whether any movement of the support foundation blocks occurred, and a theodolite was used to check that the columns of the loading rig remained vertical during the tests.

Strains were measured from the time when the precast units were stressed until the composite beam failed; mechanical "Demec" gages operating on 8 in. gage lengths were used. Locating discs were fixed to the precast concrete with sealing wax and were cast in the in situ top flange. The gage lengths were arranged in banks near each end of the beam, at the erection joints and at sections 4 ft to either side of the center-line; the gages were identical on both sides of the beam.

ANALYSIS AND DISCUSSION OF RESULTS

Theoretical working load and ultimate load of beam.—The theoretical working-load conditions have been abstracted from the consulting engineer's calculations. The following are the mid-span bending moments for which the beam was designed:

Design live load	= 26,500,000 lb in.
Road surfacing	= 3,630,000 lb in.
Additional in situ concrete	= 3,420,000 lb in.
Additional effect of removing props under edge beam	= 4,150,000 lb in.
Total effective superimposed load moment	= 37,700,000 lb in.

From the given stress conditions in the beam (that is, when part of the eventual total loss of prestress had occurred) the moment required to produce zero stress in the bottom fiber of the composite section at mid-span was calculated; it was found to be 39,400,000 lb in. This moment corresponds with jack loads of 73 kips for the arrangement adopted during the tests. If the modulus of rupture of the precast concrete is assumed to have been 800 lb per sq in. then the estimated moment to cause cracking at mid-span was 63,200,000 lb in. or 117 kips per jack.

From the stress conditions at the erection joint, the moment required to produce zero stress in the bottom fiber at that section was calculated. It was found to be 42,800,000 lb in., which corresponds to a jack load of 99 kips.

In the analysis of the results the term "working load" is used to define that superimposed load which causes zero stress in the bottom fiber of the beam at mid-span.

The ultimate strength calculations were necessary to determine the size and strength of the testing rig, and hence certain assumptions were made regarding the strength of the concrete at the time of the test. These assumptions and the calculations are given in the Appendix. From the results of the cube tests, given in Tables 1 and 2, it will be seen that a satisfactory value was assumed for the strength of the in situ concrete but that the strength of the precast concrete was under-estimated. The actual beam tested differed from the one assumed in the ultimate load calculations in that the thickness of the in situ top flange was 13 in. instead of 12 in. although the over-all depth was the same. It is not surprising, therefore, that the estimated ultimate load of the beam was on the low side.

The estimated ultimate load corresponded with a load of 248 kips per jack and the testing rig was therefore designed for a total applied load of 550 kips.

Test results.—After the three precast sections of the beam had been placed on the two end supports and the temporary supports at the joints, and had been levelled and jointed, strain gage locating discs were attached to the sides of the beam. The day before stressing was commenced the gage lengths were measured and they were checked immediately before stressing was begun. This check, and subsequent visual inspection, showed that the joint nearer the rocker support (the east joint) had opened and cracks approximately 24 in. x 10^{-3} in. wide had formed. This cracking is thought to have been produced by movements of the temporary timber supports used in the test. The other joint (the west joint) was also apparently cracked. The maximum gage reading was 2 in. x 10^{-3} in., but the crack was not visible in the roughened concrete at the joint.

At suitable stages in the stressing operation strain readings were taken at the ends of the beam, at the joints, and at mid-span. On the first day one cable was stressed and the cracks in the joints closed to a considerable extent. Slip in the anchorage, however, necessitated release of the prestress and the cracks re-opened to a greater width than before. The stressing operations extended over a period of nine days (including the weekend) owing to interruptions which were partly due to the investigation of difficulties. Measurements of the strain-gage lengths across the joints showed that although the cracks became invisible after the first two cables had been stressed, the movement at the joints continued until five or six cables had been stressed. After that the strain measurements across the joints were similar to those on adjacent uncracked con-

crete. The differences between the readings of gages across the east joint and on adjacent uncracked concrete near the soffit of the beam were as follows:

During stressing of cable No.	Closure of crack was approximately (x 10^{-4} in.)
1	180
2	9 90
3 and half 4	40
half 4 and 5	9

Friction between cables and ducts at the joint may conceivably have prevented the strain due to the closure of the crack from being spread over the whole length of the cables with the result that the stressing of the second, third, fourth, and fifth and sixth cable caused a very local loss of prestress in the cable or cables already stressed. The size of this loss cannot, of course, be estimated with any degree of accuracy.

To investigate the friction along the entire length of the cables during stressing, strain-gage lengths at each end of the beam were measured. These indicated very small losses due to friction; the strains at the stressing end and dead end being 23.0 x 10^{-5} and 22.4 x 10^{-5} with 9 cables stressed and 46.8 x 10^{-5} and 45.9 x 10^{-5} with 18 cables stressed. Each of these values is the average of four gage lengths. If the modulus of elasticity of the concrete was the same at the two ends of the beam, the loss due to friction along the whole length of the beam was only about 2%. In the design a loss of 3% from the jacking point to the mid-span section was assumed.

To check the design assumptions concerning the loss of prestress due to creep and shrinkage of the concrete and to relaxation of the steel during stressing, the readings from the four banks of gage lengths near mid-span were averaged and are plotted in Fig. 8. To convert these strains to stresses requires a knowledge of the modulus of elasticity of the concrete. Attempts to obtain this were made during stressing by measuring the shortening of the soffit of the beam and the hogging at mid-span. The variation of the cross-section of the beam and the prestress along the span complicated the calculations. The overall shortening of the soffit of the beam during the stressing of the 18 main cables was 0.62 in. and this corresponds with an estimated average stress of 2,320 lb per sq in. (it being assumed that there was no loss due to friction and no relaxation of the steel). The modulus of elasticity calculated from these figures is 5.4 x 10^6 lb per sq in. The hogging of the beam during stressing was 1 7/16 in. and this agreed well with the theoretical value calculated on the assumption that E was 5.5 x 10^6 lb per sq in. Thus it seems reasonable to accept the value of 5.4 x 10^6 lb per sq in. for converting strains to stresses and this gives the actual stress distribution on the completion of stressing as +2,700 lb per sq in. (compression) in the bottom fibers and +702 lb per sq in. (compression) in the top fibers. The design values calculated on the assumption of a 3% loss at mid-span due to friction and a loss due to relaxation of the steel during stressing, were +2,140 lb per sq in. and +765 lb per sq in. If there was no less at mid-span due to friction and no loss due to relaxation during stressing the design stresses would be +2,412 lb per sq in. and +688 lb per sq in. The agreement is still not very good and simple adjustments such as altering the modulus of elasticity do not close the gap between theoretical and actual stresses. Because of the length of time taken for the stressing operation strains due to creep in the concrete may have been a

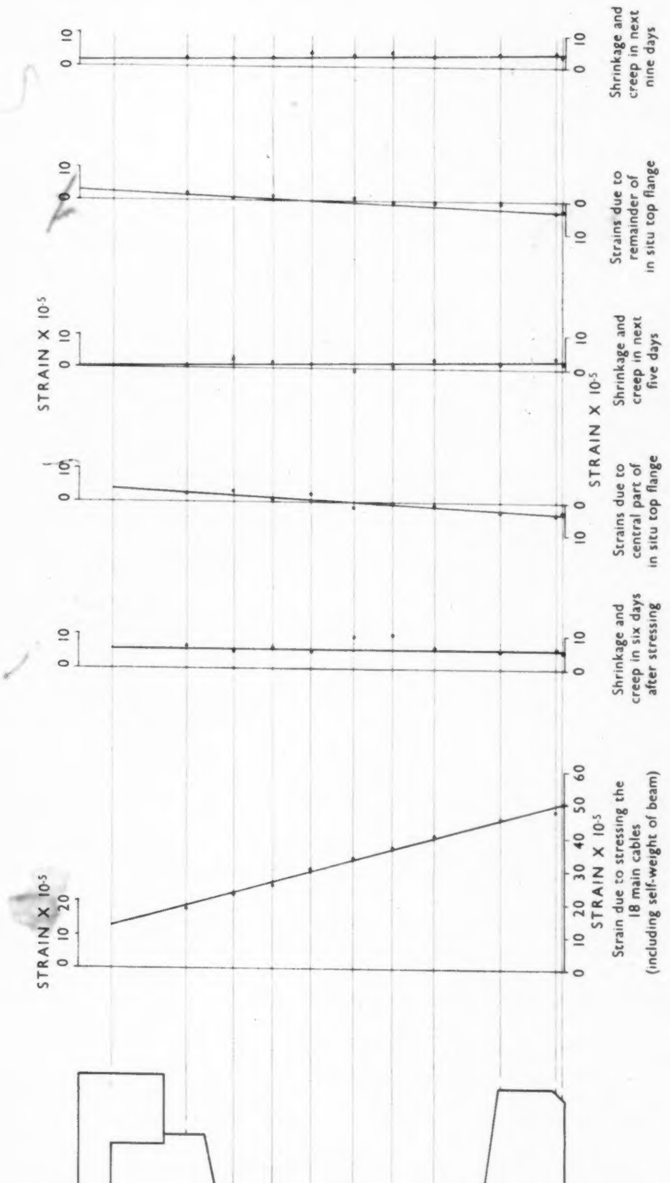

FIG. 8.—STRAINS IN THE CONCRETE AT MID-SPAN DUE TO PRESTRESSING THE 18 MAIN CABLES AND CASTING THE TOP FLANGE

significant part of the measured strain and, further, the temperature variation was 14° F during the stressing and involved large corrections. These factors, together with some error in the determination of E would account for the discrepancy, but it seems clear that the stresses on the completion of stressing were at least as high as designed.

After completion of stressing further strain measurements were taken before and after the casting of each section of the top flange. In the six days before the central section was cast the central banks of gage lengths showed an approximately uniform shortening over the whole depth of the beam owing to shrinkage and creep, the strain being $+6.5 \times 10^{-5}$. The loss of prestress due to this would be $6.5 \times 10^{-5} \times 28 \times 10^{6} = 1,820$ lb per sq in. in the steel, that is approximately 1.2% of the initial stress in the wire. The addition of the central section of the top flange produced strains of approximately -3×10^{-5} in the bottom of the beam and $+4 \times 10^{-5}$ in the top, corresponding to a modulus of elasticity of 7.5×10^{6} lb per sq in. In the five days following the casting of the central in situ flange, strains due to shrinkage and creep in the precast section were triangular in profile, varying from $+2.5 \times 10^{-5}$ in the bottom fibers to 0 in the top. Casting the end sections of the in situ flange, produced strains in the precast section at mid-span of -3×10^{-5} in the bottom fiber and $+2.5 \times 10^{-5}$ in the top, corresponding to an E of 4×10^{6} lb per sq in. Strains due to creep and shrinkage in the precast section after completion of the in situ concreting varied from $+4 \times 10^{-5}$ in the bottom fiber to $+2 \times 10^{-5}$ in the top during the first nine days. The total strain in the bottom fibers from just before casting the central in situ top flange until just before testing the beam was negligible, and the loss of prestress must have been very small. Strain gage plugs were cast into the in situ concrete but no readings could be taken before stripping of the formwork. In the nine days after completion of the in situ concrete, strains due to shrinkage of approximately 5×10^{-5} occurred.

Loading tests on the beam commenced 25 days after the completion of stressing and 13 days after the completion of the top flange. On the first day a load of 157 kips on each of the two loading points was reached, deflection gauges being read at increments of $5\frac{1}{2}$ kips and strain gages at increments of approximately 22 kips. At 134 kips per jack very fine cracks were found in the soffit of the beam near mid-span and reference to load-strain curves (Fig. 9) shows that the cracks actually commenced at slightly under 134 kips, probably at 129 or 130 kips per jack. The calculated value was 117 kips per jack.

Fig. 10, of strain gage measurements from gage lengths placed across the joints, and Figs. 11, 12, 13 and 14, of load-deflection graphs, show clearly that the east joint was opening at a load of approximately 67 kips per jack. The departure from linearity of the load-strain graphs for the east joint, north face, is quite marked at 67 kips but is not so clear for the south face. Behaviour of the west joint was similar but is less marked on the graphs. The surfaces of the beam at the joints were very rough and it was not possible to locate fine cracks by eye. A load of 67 kips per jack is considerably less than that at which the joints should have reopened but, as already suggested, this may have been due to very local loss of prestress because the joint closed during stressing.

The central deflection of the beam at 134 kips per jack was 2.7 in. and at 157 kips per jack was 3.5 in. Immediate recovery on removal of the load was to within 1/3 in.

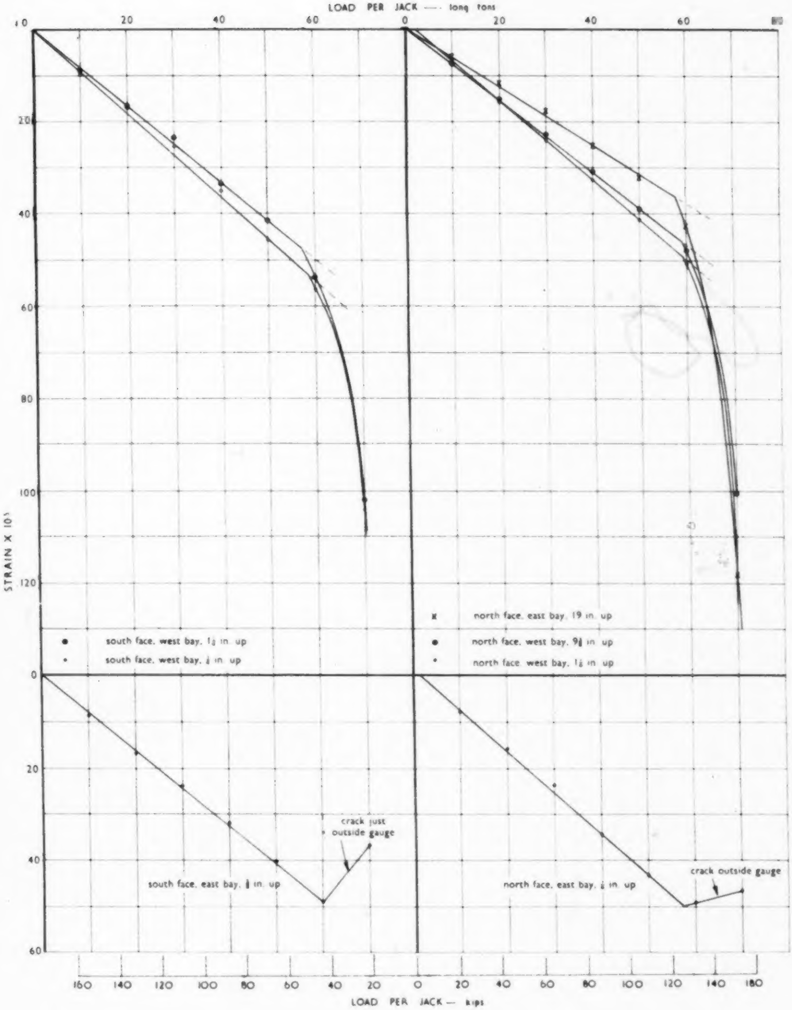

FIG. 9.—STRAIN GAUGE MEASUREMENTS NEAR THE BOTTOM OF THE BEAM NEAR MID-SPAN

Fig. 15 shows strain profiles near mid-span during the first test loading. A crack can be seen to have passed right through one bank of gages but the other bank was only partly intersected by a crack.

After a pause of half an hour a second loading was carried out to 148 kips per jack, the load being applied as quickly as possible. The only pauses were of a few seconds each as the two dial gages at mid-span were read at each increment of load (approximately $4\frac{1}{2}$ kips per jack). A similar rapid loading

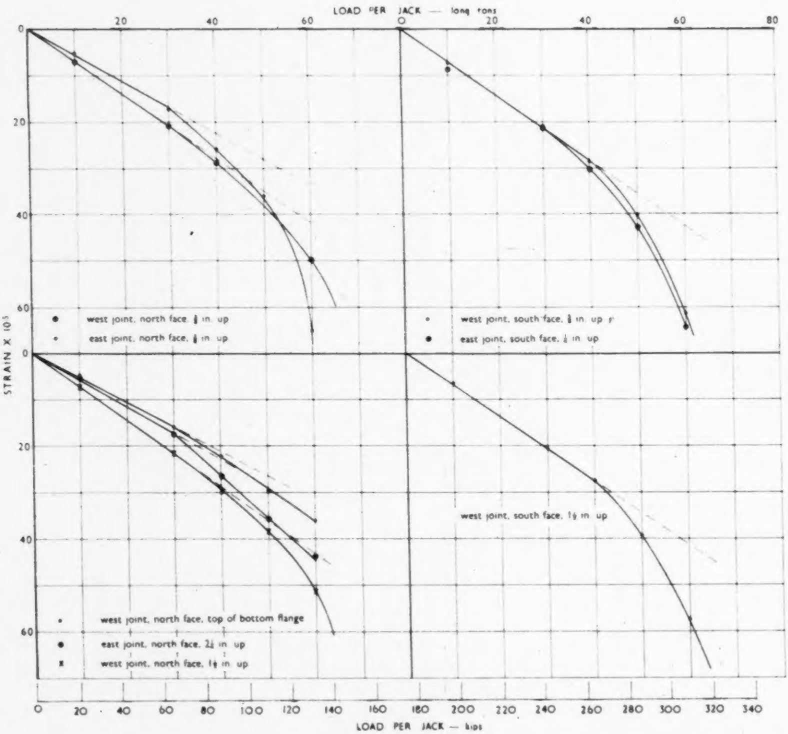

FIG. 10.—STRAIN GAUGE MEASUREMENTS NEAR THE BOTTOM OF OF THE BEAM ACROSS THE JOINTS

was carried out at the beginning of the second day's testing, the aim being to determine as accurately as possible the load at which cracks in the beam reopened, that is, the zero tension load. The speed of loading reduced the effect of creep and it can be seen from Figs. 11, 12 and 14 that the beam appeared to be stiffer under the faster loadings than under the slow first loading. The load-deflection graph (Fig. 16) obtained by averaging the readings of the second and third loadings, two dial gages being used each time, shows that cracks were reopening at 77 or 78 kips per jack. This is approximately $4\frac{1}{2}$ kips per jack

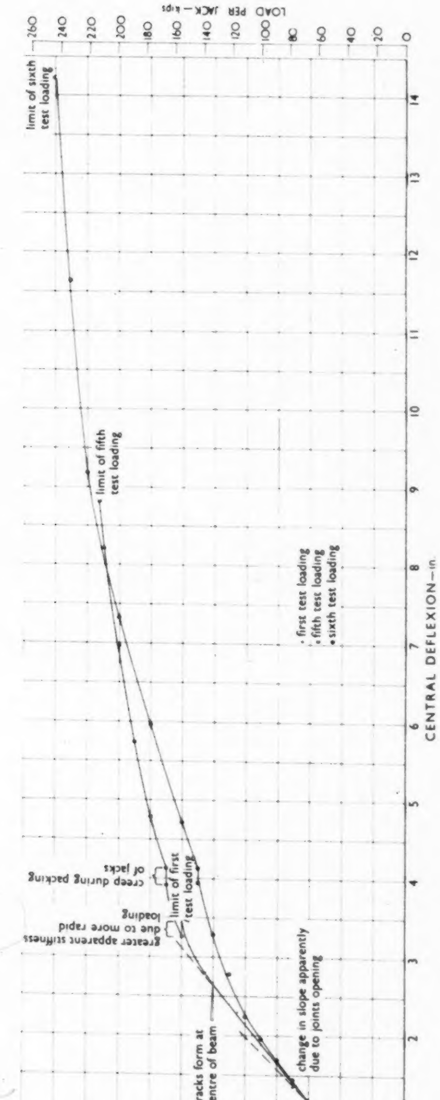

FIG. 11.—CENTRAL DEFLEXIONS: AVERAGE READINGS OF TWO VERNIER GAUGES FOR FIRST, FIFTH, AND SIXTH TEST LOADINGS

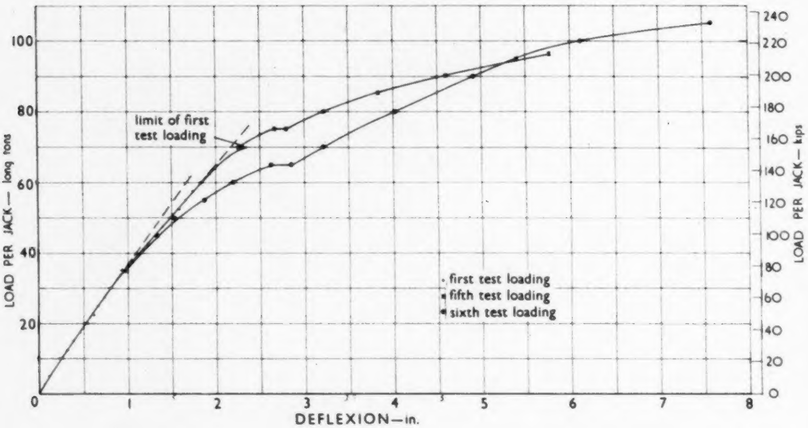

FIG. 12.—DEFLEXIONS AT 1/4 SPAN EAST: VERNIER GAUGES

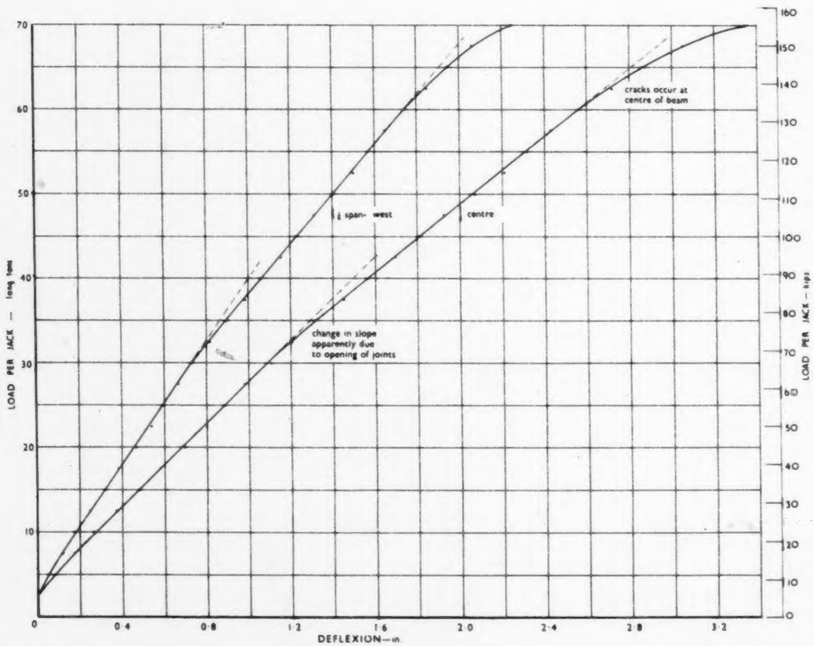

FIG. 13.—DEFLEXIONS AT CENTRE AND 1/4 SPAN WEST: AVERAGE READINGS OF TWO VERNIER GAUGES FOR FIRST TEST LOADING

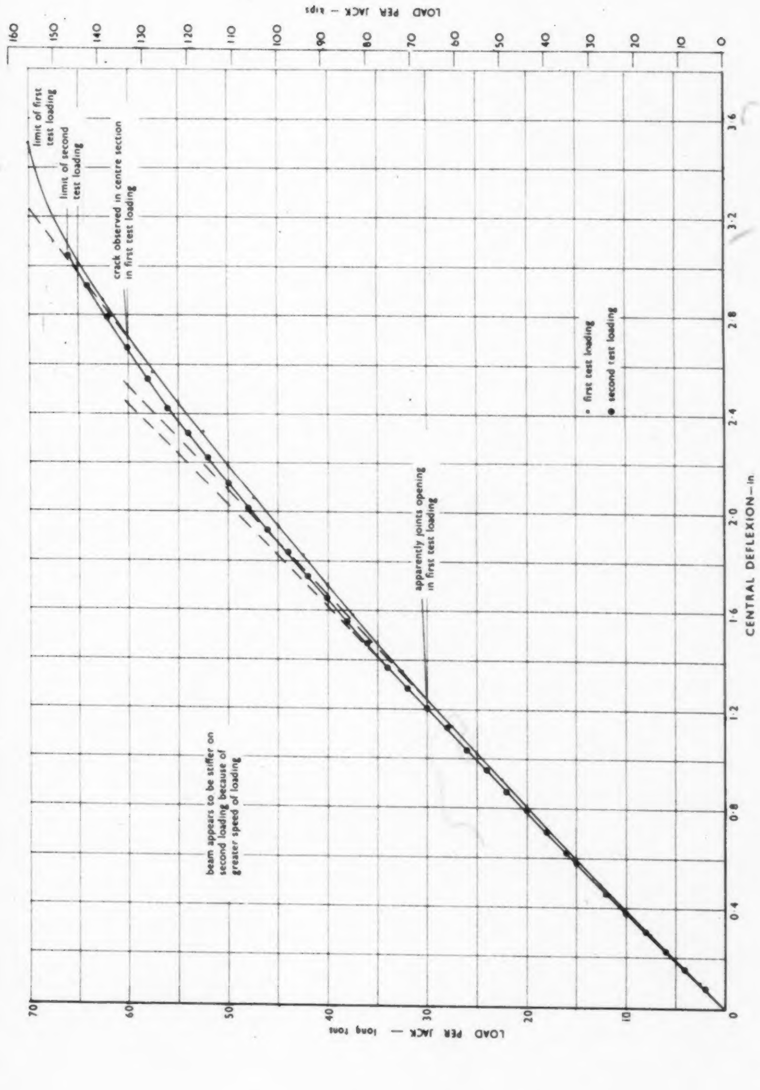

FIG. 14.—CENTRAL DEFLEXIONS: AVERAGE READINGS OF TWO DIAL GAUGES FOR FIRST AND SECOND LOADINGS

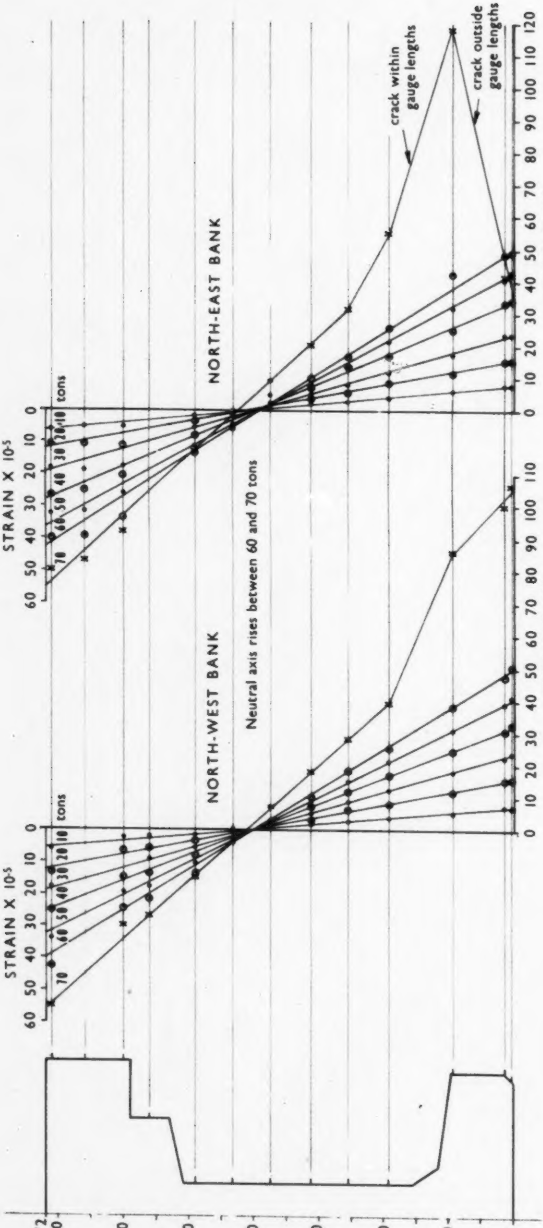

FIG. 15.—STRAIN PROFILES NEAR CENTRE OF BEAM DURING THE FIRST LOADING (NORTH FACE)

greater than the theoretical load required to reopen cracks at mid-span. It must be assumed that the joint-opening load obtained from load-deflection graphs of the first loading was slightly depressed by the effect on the graphs of creep due to the slow rate of loading. The more rapid loadings show no departure from linearity of the load deflection graphs at 67 kips, but it is impossible to say whether the departure at 77 kips is due to reopening of the joints

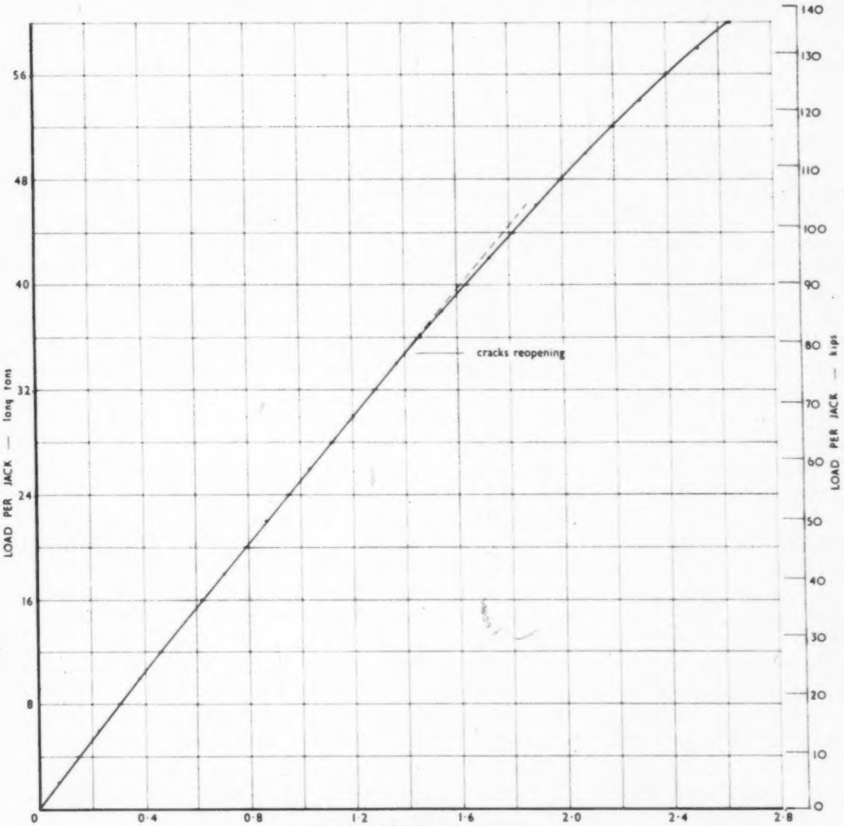

FIG. 16.—CENTRAL DEFLEXIONS: AVERAGE READINGS OF TWO DIAL GAUGES FOR SECOND AND THIRD TEST LOADINGS

or to reopening of cracks at mid-span. The initial cracking of a beam is usually visible almost immediately after it begins but the reopening of cracks on subsequent loading is extremely gradual and is difficult to locate exactly. Time did not allow further loadings to be made to obtain load-strain graphs for the joints and mid-span. The actual central deflection of the beam at 77 kips was 1.385 in. and the value calculated before the test was 1.375 in.

After a loading up to 157 kips per jack, when an oil leak necessitated removal of the load, the beam was reloaded to 157 kips per jack and then, by

approximately 11 kips increments, to 212 kips per jack. At 168 kips per jack it was necessary to pack the jacks, which were almost at the end of their travel. This operation took an hour, and the beam deflection increased 0.2 in. during this time owing to creep. At 179 kips per jack a sharp sound was heard and it was found that diagonal cracks had formed across the web between the loading points and the transverse duct holes at the construction joints (Fig. 17). At about 215 or 217 kips per jack (the loads were not exactly equal) when the total central deflection was 8.9 in. the jacks had again reached the end of their travel and it was considered too dangerous to repack at that stage. Testing was therefore stopped until a pair of electrically operated jacks with 12 in. travel, which had become available after the commencement of the tests, could be inserted in place of the 6 in. jacks with hand pumps. These jacks were loaded to 145 kips when the central deflection was approximately 4 in. and then packed and reset to give a further 12 in. travel. At 246 kips per jack the deflection was 14½ in. and the deflection increment between 236 and 246 kips was 2.6 in. For this reason it was decided to repack the jacks once more and this was carried out after reducing the load to 157 kips per jack. Although this was only 11 kips per jack higher than the previous resetting, the "cutting across the corner" of the load-deflection curve allowed sufficient packing to be inserted for the ultimate deflection of approximately 20 in. to be obtained at a load of 264 kips per jack. Failure occurred when a length of about 4 ft of the top flange at mid-span gently crushed to a depth of about 6 in. Cracks had reached and entered the precast top flange but not the in situ concrete. The depth of the compression zone would then have been approximately 19 in. and compared well with the value of 20 in. assumed in the calculation of the ultimate load (see Appendix). The shear cracks outside the loading points had increased in width and number but did not seem to influence the failure in any way. Immediately the crushing occurred the load was reduced to a nominal value. The beam was then reloaded and, as shown in Figs. 18 and 19, collapsed completely when the load again approached 264 kips. After collapse the in situ top flange at mid-span could be seen to have parted from the precast section, the vertical stirrups having failed, but this was a secondary effect of the collapse.

During the final loading a strain gage was attached to a gage length 1 in. from the top of the beam near mid-span and strains were measured. The last reading was obtained at 258 kips per jack when the total strain was approximately 0.002 but reference to Fig. 20 shows that the assumed ultimate strain of 0.003 would probably have been almost reached at 264 kips per jack. The gage length measured was actually within the length of concrete that crushed.

After failure of the beam a number of lengths of duct were cut from the beam near mid-span and examination showed the grouting to be excellent (Fig. 21).

The ultimate failing load of the beam was about 6% greater than the calculated value. Since an accuracy of within 10% is normally required for ultimate strength calculations, this agreement is perfectly satisfactory and shows that the slight discrepancies between the assumptions on which the calculations were based and what occurred in practice did not warrant a revised ultimate load calculation.

The ultimate load gave a load factor of 3.62 on the jack load required to produce zero stress in the bottom fibers. With respect to the total effective superimposed load moment given in the section "Theoretical working load and ultimate load of beam" of the analysis and discussion, the load factor was 3.78.

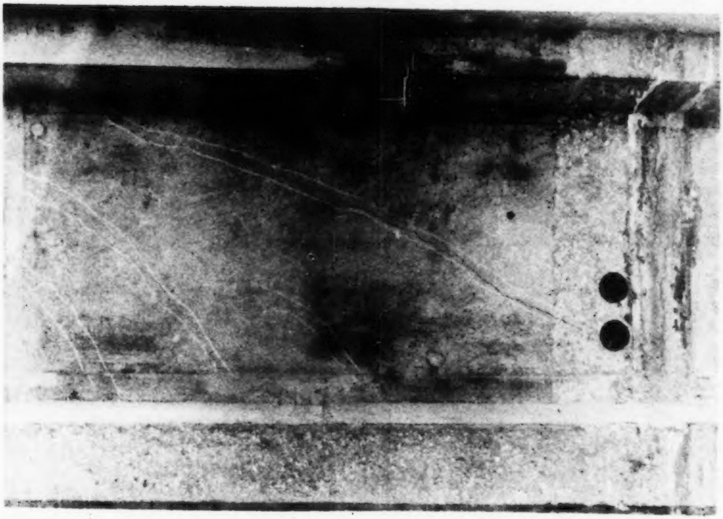

FIG. 17.—DIAGONAL CRACKS

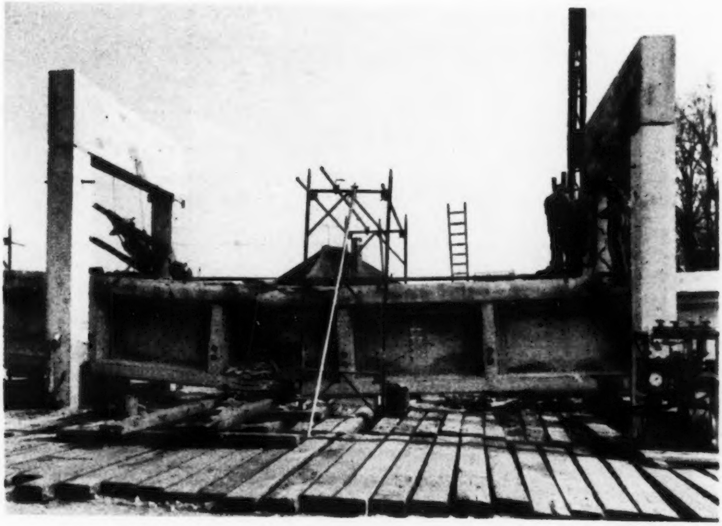

FIG. 18.—BEAM AFTER FAILURE (NORTH FACE)

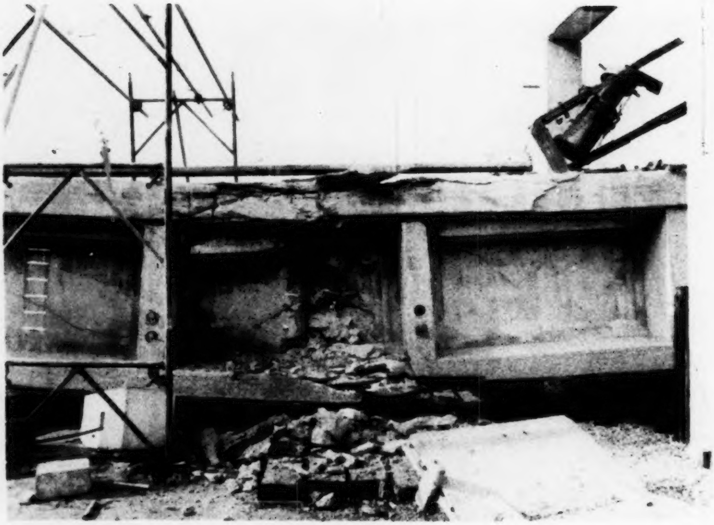

FIG. 19.—CLOSE-UP OF THE BEAM AFTER FAILURE (SOUTH FACE)

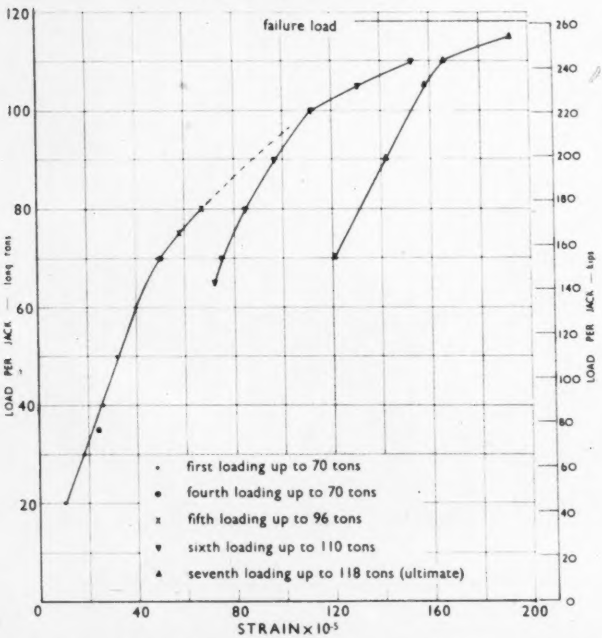

FIG. 20.—STRAIN MEASURED ON 8 IN. GAUGE LENGTH AN INCH FROM THE TOP OF THE BEAM NEAR MID-SPAN

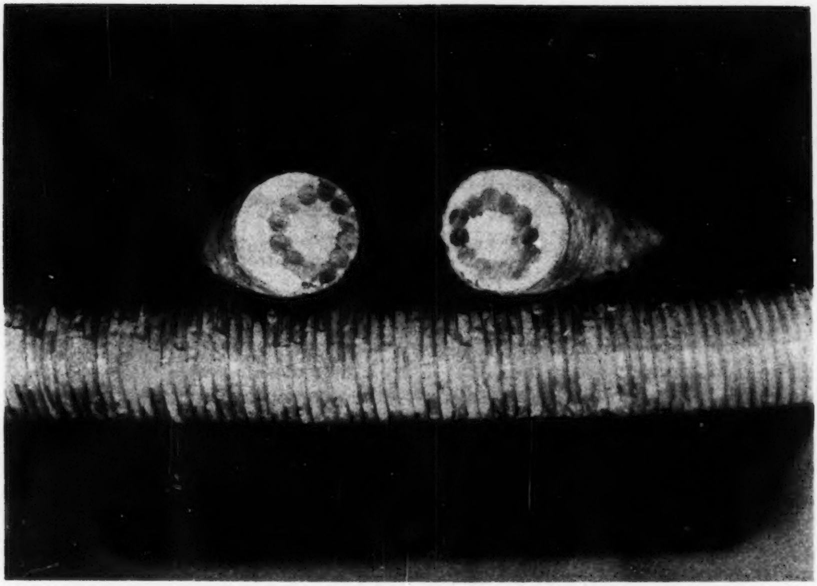

FIG. 21.—DUCTS REMOVED AFTER FAILURE, SHOWING THE GOOD GROUTING

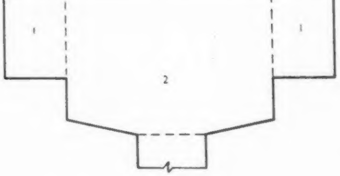

FIG. 22.—SECTIONS OF COMPRESSIVE STRESS BLOCK AT ULTIMATE LOAD

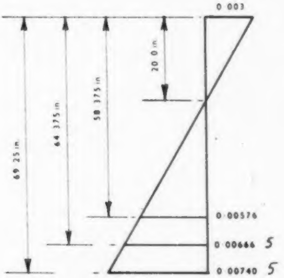

FIG. 23.—STRAIN DISTRIBUTION AT ULTIMATE LOAD

The dead load moments, abstracted from the design calculations, were as follows:

Mid-span moment due to precast concrete	= 36,200,000 lb in.
Mid-span moment due to in situ concrete	= 6,020,000 lb in.
Total dead load moment	= 42,220,000 lb in.
Hence the total dead plus superimposed load moment	= 42,220,000
	+ 37,700,000
	= 79,920,000 lb in.

Since the ultimate moment of the test beam, including the self-weight moment, was 188,220,000 lb in. the ultimate load may be represented as

2.35 (dead load + superimposed live load)

CONCLUSIONS

The tests showed that the design assumptions regarding the loss due to friction on stressing and the loss of prestress due to creep, shrinkage, and relaxation were perfectly satisfactory. The assumptions regarding the total loss of prestress could not of course be checked over any long period of time but the estimated stresses from the measured strains in the beam up to the time of the actual tests were greater than those given by the design. The higher pre-compression in the bottom fiber was shown by the fact that the load required to produce zero stress in the bottom fiber was slightly greater than anticipated.

The cracking in the joints between the precast elements which occurred before stressing was begun did not significantly affect the behaviour of the beam. Such cracks would probably heal in the actual bridge structure since some time would elapse before the top flange was cast.

The behaviour of the beam within the working-load range was excellent and agreed with the predicted behaviour. At ultimate load the agreement between the predicted behaviour and the actual behaviour was again excellent, this applied to the ultimate loads, the depth to the neutral axis, and the ultimate strain in the top fiber of the beam. The presence of a number of shear cracks originating from the holes through the web for the transverse prestress did not affect the ultimate strength of the beam. In the actual bridge transverse diaphragms will strengthen this region of the beam, and such cracks will probably not occur.

The load factor with respect to the total dead + superimposed loading was 2.35, which is adequate for a structure of this type.

ACKNOWLEDGMENTS

The Cement and Concrete Association supervised the test and carried out the analysis of the results. This was done with the full collaboration of Harry Brompton, the Ministry's Consulting Engineer; Alderton Construction Co. Ltd., the main contractors for the overpass, and Cowley Concrete Co. Ltd., who

were responsible for the manufacture of the precast beam sections and the preparation of the testing rig.

APPENDIX. ULTIMATE LOAD CALCULATIONS

Center section of the beam
Steel

Initial stress = 157,000 lb per sq in.

Final stress in main cables = 123,000 lb per sq in. at which strain = 0.00435

Final stress in secondary cables = 116,500 lb per sq in. at which strain = 0.00409

Concrete

Assumed cube strength of in situ concrete = 6,000 lb per sq in.

Assumed minimum cube strength of precast concrete = 7,500 lb per sq in.

Ultimate moment

With such a large over-all depth it is highly probably that the bond factor, F, will be unity and this is one of the basic assumptions of the analysis. The other assumptions are:

(a) The ultimate strain in the top fiber of the concrete at failure is 0.003 for the assumed cube strength of 6,000 lb per sq in. in the in situ concrete;

(b) The compressive area in the beam is divided into the sections shown in Fig. 22 and for these sections the average stress and the position of the centroid of the stress block are as given below.

Section (1) Average stress taken as $\frac{2}{3}$ x cube strength of in situ concrete. Centroid assumed at mid-depth of section.

Section (2) Concrete assumed to be uniform in this section with an average stress of 0.5 x cube strength of the precast concrete. Centroid assumed to be at a depth of 0.4 x depth to neutral axis.

The distribution of strain over the depth of the beam is shown in Fig. 23. From the strains given by this distribution and those due to the final prestress, the ultimate strains in the steel at the various levels are found to be

0.00985 at which the load per wire = 13,100 lb
0.01101 at which the load per wire = 13,350 lb
0.01175 at which the load per wire = 13,420 lb

Hence the total force in the steel is

$$12 \times 13{,}100 + 108 \times 13{,}350 + 108 \times 13{,}420$$
$$= 157{,}000 + 1{,}442{,}000 + 1{,}449{,}000$$
$$= 3{,}048{,}000 \text{ lb}$$

The compressive force in the concrete is

Section (1): 18 x 12 x 4,000 = 864,000 lb
Section (2): 30 x 19.5 x 3,750 = 2,194,000 lb
Total concrete force = 3,058,000 lb

$$\text{The centroid of the concrete force} = \frac{(864{,}000 \times 6) + (2{,}194{,}000 \times 0.8)}{3{,}058{,}000}$$
$$= 7.45 \text{ in. from top fiber.}$$

The ultimate moment is

$$M_{ult} = (157{,}000 \times 50.925) + (1{,}442{,}000 \times 56.925) + (1{,}449{,}000 \times 61.80)$$
$$= 1.749 \times 10^8 \text{ lb in.}$$

This moment is equivalent to 332,200 lb at each jacking point.

The moment is due to the self-weight of the beam is

$$M_D = 45.22 \times 10^6 \text{ lb in.}$$

which is equivalent to a load of 83,800 lb at each jacking point.

The additional load required is therefore 248,400 lb per jack.

Journal of the

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Proceedings of the American Society of Civil Engineers

DISCUSSION

ITALIAN ARCH DAM DESIGN AND MODEL CONFIRMATION[a]

Discussion by Merlin D. Copen

MERLIN D. COPEN,[2] F. ASCE.—Mr. Oberti has presented an excellent summary of the development of arch dam design in Italy. The writer concurs completely with his statement regarding the importance of "rational" dam design. There does seem to be, however, a divergence of opinion on what constitutes "rational" design. Mr. Oberti states that "the dam should be provided with a perimeter joint. . ." It has been noted, however, that most other European designers prefer not to have a perimeter joint. This practice has not, to this writer's knowledge, been used in any dams in the United States.

The author strongly supports the double-curvature, dome-like arch dam. Most of the dams illustrated in this paper seem to have extensive downstream curvature. It has been the experience of the designers in the U. S. Bureau of Reclamation, Dept. of Interior (USBR), that the principal advantage of double-curvature is to reduce or eliminate areas of tensile stress. The existance and magnitude of tensile stresses are largely determined by the physical properties of the damsite. In a wide valley, for example, the tendancy for tensile stress is much greater than in a very narrow site. "Rational" design might then indicate either radical or moderate double curvature, depending on the site conditions.

The areas in an arch dam where tensile stresses might normally be expected to occur are as follows: At the abutment extrados in the lower one third of the dam; at the crown intrados in the lower portion of the dam; at the downstream face in the vicinity of the crown cantilever near the top of the dam; and at the upstream face of the crown near the bottom of the dam. Of these tensile areas, the two that benefit directly from double curvature are the crown area near the top of the dam on the downstream face, and the vicinity near the base of the dam on the upstream face in the area of the crown. Mr. Oberti seems to confirm this when he states with regard to the Osiglietta Dam that, "The tests confirmed the reduction, through second curvature, of the tensile stresses along the cantilevers, and this proved conclusively the contribution afforded by the cantilevers because of their arched shape." This writer finds it necessary to very carefully proportion the "second curvature" to reduce the tensile stress in both of these areas. By permitting excessive downstream curvature in the upper part of the dam, the tensile stress near the bottom may actually be increased.

Mr. Oberti implies that it is not possible to compute by analytical methods, the stresses in a double-curvature arch dam. The USBR has been able to analyze all types of arch dams, including double-curvature dams, by the trial load method of analysis. While this may once have been very tedious, the application of electronic computers to assist in the computations has made the

[a] March, 1960, by Guido Oberti.
[2] Engr., Bur. of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

solution entirely practical. Furthermore, it is anticipated that still more reduction in both labor and time is possible as the analysis is more completely programed for electronic processes.

Regarding movements of foundation rock, it is well known by all designers of arch dams that practical and economic factors greatly limit the available information on the character of the foundation material. With only a meager knowledge of the rock properties and the ability to correct many of the deficiencies of the foundation material (grouting and so on), it is doubtful if the accuracy obtained by the use of models is superior to that ascertained by analytical means.

The writer has observed at various times the implication by other writers that the choice of horizontal and vertical elements is not adequate to completely determine the stresses in arch dams. If all of the forces and moments are computed in the analysis by use of the proper design loads and all movements accounted for, stresses may be computed in any direction and isostatic lines easily determined. On the other hand, it is impossible to choose elements accurately in the direction of isostatic lines, since these cannot be known until an analysis is completed.

Mr. Oberti states that "The analysis confirms that, in the elastic stage, the basic resisting elements are the arches, whereas the cantilevers offer a comparatively limited contribution owing to the curvature ratios of the structure." This is entirely true if the site is a narrow V-shaped canyon. If, however, the site is a wide V or U shape, the cantilevers will be found to be the principal resisting elements. The importance of the arches and cantilevers varies between these extremes, depending on the shape of the site and the relative flexibility of the elements.

This writer has not found any dam where conditions of non-symmetry, abutment shape, or existence of joints have made it impossible to determine stresses by analytical means. It is generally found where assumptions are necessary for analytical solutions, that these assumptions, or others, are also required for model analysis.

The writer has been very much impressed with the high quality work that has been produced by Mr. Oberti and his associates, not only in model analysis but also in the design and construction of impressive arch dams. It is recognized that under some conditions an analytical solution may not be possible for all design problems and the use of models could be essential. While the USBR uses the trial load method almost exclusively, it is constantly seeking ways to improve design techniques and the safety and efficiency of its structures. In this search, the use of models has not been ignored, as evidenced by the model studies of Stevenson Creek Dam as early as 1927, followed by Gibson Dam, Hoover Dam, and Glen Canyon Dam. It is the writer's opinion that, with the interest now indicated in arch dam design, improved design methods will evolve, resulting in more competent and efficient arch dams.

THE IMPACT OF COMPUTERS ON ENGINEERING EDUCATION[a]

Discussion by William A. Boe

WILLIAM A. BOE,[7] M. ASCE.—The paper by Mr. Fisher is well worth reading. Digital computers have had and will have a great effect on civil engineering. Problems that took days to solve, such as the stresses in suspension bridges due to suspender deflections, can be solved in a matter of minutes by computers. In the architectural and civil engineering offices, we find today that many engineering designs are being pulled out of graphs and tables that were tabulated by the digital computer. There will be even more of this in the future. What will become of the design engineer? Is he merely to be a "handbook" engineer? If he is, it would be more economical to hire one with no engineering training.

This writer firmly believes that there is a need for the design engineer and that he must be able to fulfill the following requirements:

1. He must be able to apply his theoretical knowledge to problems that have not been set up in the computers. The digital computer is expensive to operate and many problems can be worked more economically on hand computers. A problem that has not been programmed for the computer can usually be worked out faster by ordinary methods.

2. He must be able to use his imagination and come up with practical solutions to problems. He must be able to visualize new techniques in building, new materials with which to make his design, and to use theories that fit his design. Not all beams are supported on knife edges and all loads are not uniform or pin pointed. The theories are only as good as the assumptions, no matter how they are computed. The one comment heard most by this writer is that engineers have no imagination. Colleges and universities must assume much of this responsibility. Many professors "spoon-feed" the student instead of allowing him to work out the solutions by himself. The student's imagination must be kindled.

3. He must be able to supervise the drawing of plans and to write specifications so that the structure can be built. If the structure cannot be built, the design solution is worthless, no matter how it has been computed. A partner of a large engineering firm told this writer that he can always find a structural analyst, but that it is difficult to find an engineer who can turn out a good set of plans.

Colleges and universities should take a long look at what they want their civil engineering graduates to accomplish through their education. Very few graduates will have direct contact with programming-design calculations, while

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[a] April, 1960, by Gordon P. Fisher.

[7] Structural Engr., Green Engrg. Co., Middleton, Wis.

many will be called upon to draw up plans, write specifications, supervise construction, and make design computations that require no digital computers.

It was pleasing to this writer to find that Mr. Fisher did not believe that the college curriculum had to be changed to include computer design, but that an awarness of its use should be included and encouraged. All engineering graduates should have a good fundamental knowledge of the mathematics involved in computer computations.

REVIEW OF RESEARCH ON COMPOSITE STEEL—CONCRETE BEAMS[a]

Discussion by W. J. Jurkovich

W. J. JURKOVICH,[3] M. ASCE.—Mr. Viest states that "A second development deserving a brief mention is the current attempts to improve the natural bond through the use of epoxy resins." Brief mention is all that these attempts deserve presently, as far as the preliminary tests made by the California Division of Highways-Materials and Research Laboratory reveal.

The tests consisted of jacking against a 12 in.-ky-× 12 in.-ky-× 6 in. block that was cast on the flange of a steel beam. Epoxy resin was applied to the flange before casting the block. The shear stress at failure was 100 psi. Failure was in the concrete about 1/2 in. to 1/4 in. above the steel flange. No "tie downs" were used in the test and as a result, no mechanical resistance to uplift existed. Without question, mechanical "tie downs" are required for any method used to provide composite action.

Plans are being made (August, 1960) to run a full scale test using 2 1/2 in. round studs at 24 in. centers and epoxy applied to 50% of the flange before casting the deck slab. No preparation of the flange, such as sandblasting, will be done. The test will be made to determine if composite action can be maintained to ultimate load. It is expected that these tests may deserve more than a brief mention in the future.

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[a] June, 1960, by Ivan M. Viest.

[3] Senior Bridge Engr., Calif. Div. of Highways, Sacramento, Calif.

ERRATA

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Proceedings of the American Society of Civil Engineering

January, 1960

p. 5. Eq. 6(b) should be written as follows:

$$M_{BA} = M_{FBA} - k_{BA}\,(1 + \beta')\,\frac{\Delta_A}{L_{AB}} + k_{BA}\,(1 + \beta')\,\frac{\Delta_B}{L_{AB}} + \beta k_{AB}\,\theta_A + k_{BA}\,\theta_B \quad \text{(6b)}$$

May, 1960

p. 113. The line preceding Eq. 3 and Eq. 3 should be changed as follows: due to the strains $\epsilon_x = u_x$, $\epsilon_y = u_y$, $\gamma = u_x + v_x - \frac{2c}{ab}\,w$, the following expression

$$\begin{aligned} U_M &= \frac{h}{2} \iint \left(\sigma_x\,\epsilon_x + \sigma_y\,\epsilon_y + \tau_y\right) dx\,dy \\ &= \frac{Eh}{2\left(1 - \mu^2\right)} \iint \left(\epsilon_x^2 + \epsilon_y^2 + 2\mu\,\epsilon_x\,\epsilon_y + \frac{1 - \mu}{2}\,\gamma^2\right) dx\,dy \\ &= \frac{Eh}{2\left(1 - \mu^2\right)} \iint \left[u_x^2 + v_x^2 + 2\mu\,u_x\,v_y + \frac{1 - \mu}{2}\left(u_y + v_x - 2\alpha\,w\right)^2\right] dx\,dy, \quad (3) \end{aligned}$$

p. 114. In Eq. 7(a) place a bracket before the term X''_u and a bracket after the term Y''_u.

p. 116 and 117. In Eqs. 21 and 22 change all γ's to θ's.

July, 1960

p. 141. In line 8 change "low-allow" to "low-alloy."

p. 144. In Table 1, over Cols. 3 through 6, add "in ksi" to the heading

p. 148. In line 2 change "Fig. 4(c)" to "Fig. 4(d)."

p. 151. In line 15 change "routing" to "routine."

p. 156. In the caption for Fig. 17 insert a parentheses after STUB COL.

p. 161. The two figures that go to make up Fig. 23 should be interchanged with the captions retaining their present positions on the page.

p. 162. In line 4 change the sign $\leq$ to $\geq$.

p. 163 and 165. These two pages should be interchanged.

p. 170. In item 13 change the term L/4 to L/r.

p. 172. In line 4 add a single dot over the term ϵ_{pl}.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184(WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330(CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436(ST4), 2437(ST4), 2438(HY4), 2439(SM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445(EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454(EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478(HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487(CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502(EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511(EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520(PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529(HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580(EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596(HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

c. Discussion of several papers, grouped by divisions.